

# Long-Term Evolution of a Long-Term Evolution Model

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## ABSTRACT

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This paper reviews the 25-plus year history of significant developments of the GENESIS shoreline response model. Topics discussed are line sources and sinks of sand, representation of shore-normal structures including natural sand bypassing, wave transmission by and shoreline response to shore-parallel structures, seawalls, migrating longshore sand waves, seasonal variation by cross-shore sand transport, sand transport due to tidal and wind-generated currents, preservation of the regional shape of the shoreline, and the interaction between the beach berm and the dunes behind it. Such developments have been done in a consistent way, based on thorough literature reviews, beta testing, comparison to beach behavior, and quality control. The challenges have been not only to represent the features themselves, but to be consistent to the basic assumptions of shoreline modeling theory. Through these added capabilities, GENESIS has evolved to meet the challenges of modern, multi-scale, long-term coastal engineering applications.

**ADDITIONAL INDEX WORDS:** *Shoreline change, shoreline response, Cascade, GENESIS, beach fill, groins, detached breakwaters, jetties, SBEACH.*

## INTRODUCTION

Since the 1970s, various types of numerical models have been developed for engineering applications in analyzing and predicting coastal morphological evolution on yearly to decadal time scales, as summarized in Figure 1 (see, also, Hanson *et al.*, 2003). Each model represents the complex processes from a certain standpoint depending on the nature of the problem and study objectives. Model types range from detailed, micro-process based two-dimensional and three-dimensional models (de Vriend *et al.*, 1993; DELTARES, 2007) that calculate changes of nearshore morphology over a specified area to more engineering-office oriented one-dimensional shoreline response (1-line) models (*e.g.*, Hanson and Kraus, 1989; Steetzel and de Vroeg, 1999) and beach profile change models (*e.g.*, Swart, 1975; Kriebel and Dean, 1985; Larson and Kraus, 1989; Nairn and Southgate 1993; Steetzel, 1993). The more detailed and numerically intensive models based on micro-processes can simulate many interactions of beach and dune response to hydrodynamic forcing at local scale. However, hand-in-hand with a micro-scale description is a practical restriction of incapability to simulate large areas over longer time periods and many project alternatives because of extensive run-time and limitations in input forcing information. Many beach nourishment programs have a 50-year program duration, and

many structures designed to stabilize beaches, such as seawalls, can remain functional longer than 50 years. Accordingly, there is a strong need for models that are capable of reliably, robustly, and rapidly calculating coastal evolution over decades for the evaluation of many planning and engineering alternatives.

The need to calculate long-term shoreline change and compare performance of numerous engineering alternatives over long spatial extents and time frames has led to a wide use of the 1-line (shoreline response) models, which have proven their value successfully in a wide range of projects. Among these 1-line models, GENESIS (Hanson and Kraus, 1989) has likely been applied more than any other model of its kind, exceeding installation at more than 1,000 sites worldwide (Figure 2). The objective of this paper is to present the evolution of GENESIS from a traditional site-specific 1-line model in conformity with the original theory of Pelnard-Considère (1956) that has been gradually extended and improved to be a generalized system capable of describing almost arbitrary combinations of coastal structures and beach fills, cross-shore transport, tide- and wind-driven currents, wave diffraction from multiple structures, regional depth contours, and many other features not accounted for in the original formulation. Many of these features are summarized in this paper.

## BASIC RELATIONSHIPS

The history and basic assumptions of 1-line theory, with the line taken to represent the shoreline, are discussed by Hanson and Kraus (1989). In a 1-line model, longshore sand transport is

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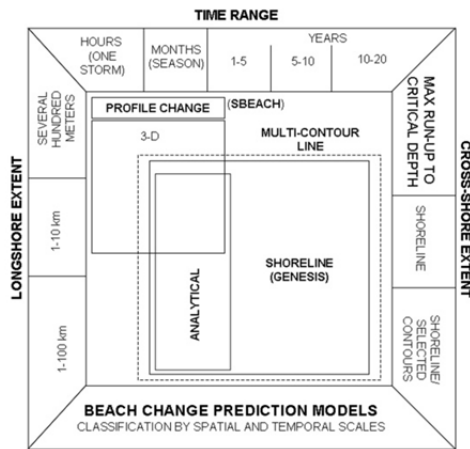


Figure 1. Spatial and temporal scales and modeling approaches (after Hanson and Kraus, 1989).

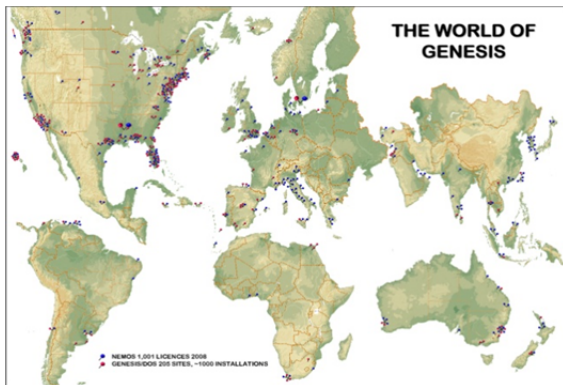


Figure 2. Known locations of GENESIS installations, 2008.

assumed to occur uniformly over the beach profile from the berm height  $D_B$  down to the depth of closure,  $D_C$ . By considering a control volume of sand (Figure 3) balanced during an infinitesimal interval of time and neglecting the cross-shore transport, the following differential equation is obtained,

$$\frac{\partial y}{\partial t} = -\frac{1}{D} \frac{\partial Q}{\partial x} \quad (1)$$

where  $y$  = shoreline position,  $t$  = time,  $D = D_B + D_C$  total height of control volume,  $Q$  = longshore sediment (sand) transport rate, and  $x$  = space coordinate along the axis parallel to the trend of the shoreline. To solve Eq. (1), it is necessary to specify an expression for the longshore sand transport rate. A general expression for this rate in agreement with several predictive formulations is,

$$Q = Q_o \sin 2\alpha_b \quad (2)$$

where  $Q_o$  = amplitude of longshore sand transport rate, and  $\alpha_b$  = angle between breaking wave crests and shoreline. A wide range of expressions exists for the amplitude of the longshore sand transport rate, mainly based on empirical results. For example, the SPM (1984) gives the following equation,

$$Q_o = \frac{1}{16} H_b^2 C_{gb} \frac{K_1}{(\rho_s / \rho - 1)(1 - \lambda)W} \quad (3)$$

where  $\rho$  ( $\rho_s$ ) = density of water (sand),  $H_b$  = breaking wave height,  $C_{gb}$  = wave group velocity at the break point,  $K_1$  = non-dimensional empirical coefficient,  $\lambda$  = porosity of sand, and  $W$  = a numerical factor ( $1.416^{5/2}$ ) necessary to convert from significant wave height to root-mean-square height in conformance with the empirical verification of the factor  $K_1$  (see Komar, 1998, Chapter 9).

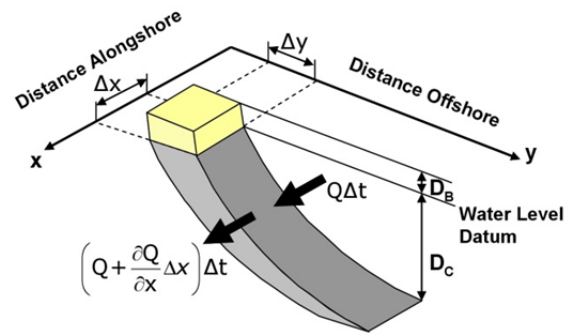


Figure 3. Definition sketch for shoreline change calculation according to 1-line theory.

## SELECTED FEATURES OF GENESIS

The following subsections highlight original contributions made to improve the capability of GENESIS for supporting practical coastal engineering applications.

### Line Sources and Sinks

In GENESIS, Eq. (1) was extended to include also line discharges of sediment representing a source or sink of sand on the shoreline following the work of Kraus and Harikai (1983),

$$\frac{\partial y}{\partial t} = -\frac{1}{D} \left( \frac{\partial Q}{\partial x} - q \right) \quad (4)$$

where  $q$  = volume of sand per unit width of beach added to beach, and which can be time dependent (e.g., river sediment discharge or beach nourishment).

### Transport near Structures (Diffraction)

Similarly, Eq. (3) was at an early stage modified to include the effect on  $Q$  from longshore gradients in wave height (Ozasa and Brampton, 1980; Kraus and Harikai, 1983). By combining Eqs. (2) and (3) and introducing the gradient effect Eq. (5) is obtained,

$$Q = \frac{H_b^2 C_{gb}}{8(\rho_s / \rho - 1)(1 - \lambda)W} \left( \frac{K_1 \sin 2\alpha_b}{2} - \frac{K_2 \cos \alpha_b}{\tan \beta} \frac{\partial H_b}{\partial x} \right) \quad (5)$$

where  $K_2$  = non-dimensional empirical constant, and  $\tan \beta$  = average bottom slope from shoreline to depth of active longshore sand transport. The contribution from the  $K_2$ -term is usually much smaller than that from the  $K_1$ -term, except in the vicinity of structures where diffraction can produce a substantial variation in breaking wave height (Kraus and Harikai, 1983). Figure 4 illustrates the effect of the  $K_2$ -term down drift of a groin or in the lee of a detached breakwater, where panel b) gives a more realistic description of the shoreline response, especially down drift of the groin.

### Jetties and Groins

Jetties and groins, as shore-normal structures, interrupt the longshore transport of sand. GENESIS was formulated to represent macro-scale (visible) properties of shore-normal structures. This work is summarized by Kraus *et al.*, (1994). Of 27 parameters that were identified to possibly influence the response of the shoreline to shore-normal structures, for a particular site it was concluded that three non-dimensional parameters exert decisive control: structure permeability, ratio of net to gross longshore sand transport rate (which varies between 0 and 1), and bypassing ratio defined as the depth at the groin tip to average deepwater wave height. Some aspects are discussed next.

**Sand bypassing.** In GENESIS, two types of sand movement past a shore-normal structure are simulated. One type is around the seaward end of the structure, called bypassing, and the other is through and over the structure, called sand transmission. Bypassing is assumed to take place if the water depth at the tip of the structure  $D_G$  is less than the depth of active longshore transport  $D_{LT}$ . This depth represents the time-dependent depth out to which sediment is transported alongshore, as opposed to

the depth of closure  $D_C$ , which may be regarded as an integrated measure over several years. In GENESIS, the calculation of  $D_{LT}$  is based on (Hallermeier, 1983).

Because the shape of the bottom profile is known from an assumed equilibrium ( $y^{2/3}$ ) profile shape (Dean, 1977),  $D_G$  is determined from knowledge of the distance between the tip of the structure and the location of the shoreline. However, because structures are located at grid cell walls between two calculated shoreline positions in GENESIS, this depth is not unique. In GENESIS the up-drift depth calculated at each time step is used.

To represent sand bypassing, a bypassing factor  $BYP$  is introduced and defined as,

$$BYP = 1 - \frac{D_G}{D_{LT}}, \quad (D_G \leq D_{LT}) \quad (6)$$

implying a uniform cross-shore distribution of the longshore sand transport rate. If  $D_G \geq D_{LT}$ ,  $BYP = 0$ . Values of  $BYP$  thus lie in the range  $0 \leq BYP \leq 1$ , with  $BYP = 0$  signifying no bypassing, and  $BYP = 1$  signifying that all sand can potentially pass the position of the structure. The value of  $BYP$  depends on the wave conditions at the given time step, since  $D_{LT}$  is a function of the wave height and period (Gravens and Kraus, 1989).

**Sand transmission.** A permeability factor  $PERM$  is analogously introduced to describe sand transmission over, through, and landward of a shore-connected structure such as a groin. A high (in relation to the mean water level), structurally tight groin that extends far landward so as to prevent landward sand bypassing is assigned  $PERM = 0$ , whereas a completely "transparent" structure is assigned the value  $PERM = 1$ . Values of  $PERM$  thus lie in the range of  $0 \leq PERM \leq 1$  and must be specified through experience and judgment of the modeler based upon, for example, the structural characteristics of the groin (jetty, breakwater), its elevation, and the tidal range at the site. Aerial photographs are often helpful in estimating a structure's amount of void space (hence  $PERM$ ) in relation to other structures on the model grid. The optimal value of  $PERM$  for each structure must then be determined in the process of model calibration.

With the values of  $BYP$  and  $PERM$  determined, GENESIS calculates the total fraction of  $F$  of sand passing over, around, or through a shore-connected structure as (Hanson and Kraus, 1989)

$$F = PERM(1 - BYP) + BYP \quad (7)$$

This fraction is calculated for each shore-connected (groin-type) structure defined on or at the boundaries of the grid.

### Detached Breakwaters and Wave Transmission

Wave transmission is a leading parameter determining the response of the shoreline to detached breakwaters, reefs, and spurs attached to jetties. Thus, the capability of representing wave transmission was introduced early in GENESIS development (Hanson *et al.*, 1989). Hanson and Kraus (1990) examined 14 parameters that might control the response of the

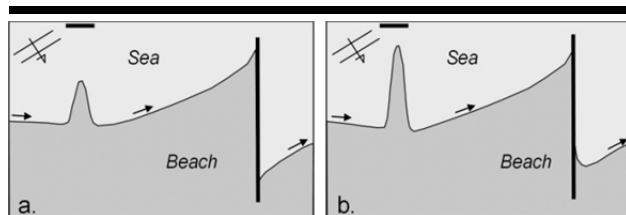


Figure 4. Qualitative effect of  $K_2$ -term in the GENESIS transport relation. Panel a)  $K_2 = 0$ , b)  $K_2 > 0$ .

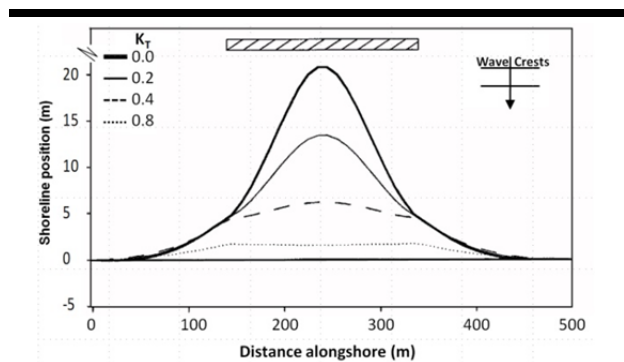


Figure 5. Shoreline response inside of a detached breakwater as a function of wave transmission (From Hanson and Kraus, 1989).

shoreline to a single transmissive shore-parallel structure. They concluded that three non-dimensional parameters were decisive in determining shoreline response: wave transmission, length of the structure divided by the average wavelength at the structure, and the average wave height divided by the depth at the structure on the equilibrium profile.

Figure 5 shows the result of simulations with GENESIS for a single detached breakwater for different values of the wave transmission coefficient  $K_T$ , which has the range  $0 \leq K_T \leq 1$ . As expected, greater wave transmission results in a smaller salient or seaward growth of the shoreline. Studies with GENESIS have shown that a single salient or tombolo will form if the diffraction sources (tips) of a detached breakwater are relatively close, and dual salients will form if the diffraction sources are relatively distant as compared to the width of the surf zone (Hanson and Kraus 1990).

The wave transmission predictive capabilities implemented into GENESIS were applied in a study for Grays Harbor, WA. The entrance to Grays Harbor is bounded on both sides by rubble mound jetties. The effectiveness of the North Jetty has decreased as a result of subsidence and deterioration. Construction of a submerged spur off the North Jetty has been proposed. GENESIS was being applied to determine if the proposed spur would be beneficial. In the study it was found that a description with a fixed value representing wave transmission would not generate adequate predictions for a site with large tidal range (about 2 m in at Grays Harbor). Based on a wave analysis, it was concluded that waves produced different responses of the shoreline to the structure based upon tide level, which had to be included in the GENESIS wave calculations. To improve the predictive capability, published empirical formulas for the wave transmission coefficient as a function of different structural and wave parameters were incorporated into the model to calculate time-dependent wave transmission and shoreline response (Wamsley and Hanson, 2002; Wamsley *et al.*, 2002; Wamsley *et al.*, 2003). Simulations for different structural configurations and wave climates demonstrated the functional utility of the time-dependent wave transmission on shoreline response predictions. Results indicated that variable wave transmission is of particular importance for submerged and emergent near surface structures.

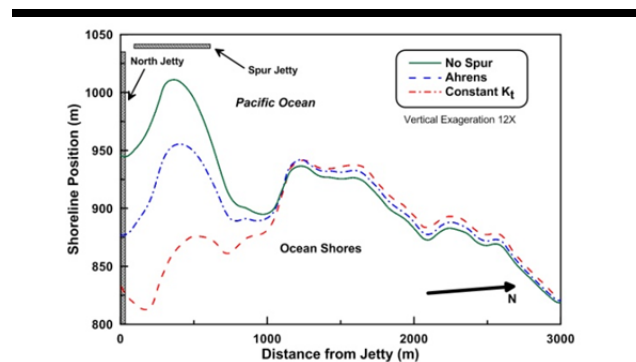


Figure 6. Predicted shoreline positions at Grays Harbor, WA. In the Ahrens case, time-dependent  $K_T$ -values were calculated. (From Wamsley *et al.*, 2002)

Thus, in GENESIS it is possible to specify for each structure either a constant  $K_T$ -value or a time-dependent  $K_T$ -value. If the variable- $K_T$  option is selected, water level is read from an input file at a specified input time interval. For each structure, the user gives geometric properties (crest height and width, slopes on seaward and landward sides, and median rock size) and may select between the calculation methods of Ahrens (2001), Seabrook and Hall (1998), and d'Angremond *et al.* (1996). The method selected should be based upon structure type and configuration. The three Wamsley references above provide guidance on this selection procedure. Based on model calibration and verification in addition to a series of sensitivity tests it was concluded that the Ahrens (2001) method considered the more realistic one. As an illustration, Figure 6 shows the difference between the Ahrens (2001) method and one using a constant value of wave transmission. The variable  $K_T$  formulation produced as much as 70 m more shoreline advance behind the spur than predicted with constant  $K_T$ . The primary reason for the difference is the sensitivity of the prediction to water level and incident wave height, acting together with directionality of the wave climate.

### Seawalls

Stretches of coast experiencing chronic erosion may need armored shorelines, especially for areas with landward infrastructure such as roads and utilities. Thus, it was early on found necessary to represent these structures in GENESIS (Hanson and Kraus, 1985). The presence of a seawall was represented as a constraint on the solution of the basic relationship (Eq. 1). This constraint was formulated on the same level of idealization as the 1-line concept. Thus, wave reflection, settling, flanking or possible collapse were not considered (but are presently being revisited). The representation of the seawall was formulated to be consistent with the basic assumptions underlying the 1-line model. The first principle was that sand volume must be conserved, meaning that there can not be a net gain or loss of sand from an area in contact with a seawall. This may sound obvious, but previous formulations of this constraint (e.g., Hashimoto *et al.*, 1971; Ozasa and Brampton, 1980;

Tanaka and Nadaoka, 1982) do not appear to have fully conserved sand volume. The second principle was that even if there is no sand in front of the seawall, the potential transport rate should be adjusted such that the direction of sand transport alongshore is preserved. Figure 7 demonstrates a hypothetical case where a divergence point of sand transport is located half-way between two groins. A seawall is located a small distance landward of the shoreline at this location. As a result of the divergence, the beach initially erodes, gradually exposing the seawall. As the shoreline reaches the seawall, it cannot recede further. Thus, the potential erosion of an exposed cell is transferred to the adjacent cell in the direction of transport, i.e., away from the divergence point. If the entire seawall is exposed, erosion occurs in the cells just adjacent to the structure on either side. The example shows that the constraint provides realistic shoreline evolution in front of as well as adjacent to a seawall.

### Longshore Sand Waves

Longshore sand waves are macro-morphologic features that maintain form while migrating along the shore with speeds on the order of kilometers per year, a collective movement of sand as discussed by Sonu (1968). Such sand waves can dominate shoreline evolution by causing both apparent long-term erosion and accretion seemingly unrelated to the calculated or estimated net and gross longshore transport rates. A study of the phenomenon and an attempt to reproduce these features with GENESIS was stimulated by observations made of longshore sand waves at Southampton, Long Island, New York. Hanson *et al.* (1996), stimulated by the work of Thevenot and Kraus (1995). Three possible mechanisms hypothesized to maintain and translate longshore sand waves were explored: wave asymmetry, form advection, and surf-zone contraction. All mechanisms were implemented within the framework of GENESIS.

To validate qualitatively the preliminary approach to modeling longshore sand wave migration, the advective form method was applied to the situation at Southampton Beach. Here, eleven sand waves present in the early 1990s were

identified from aerial photographs (Thevenot and Kraus 1995). The longshore sand waves had an average length of 0.75 km and amplitude of about 40 m. Their average migration speed was reported to be 0.35 km/year over the simulation period from September 1991 to December 1992. Their speed was found to vary seasonally with the longshore sand transport rate, and the migration speed of these organized forms was only a few percent of the anticipated speed of the individual sand grains. Results of the simulation are shown in Figure 8. Comparisons between observed and calculated longshore sand waves indicated that their movement could be modeled by the 1-line compatible method based on advective form approach. However, the temporal and spatial variation in diffusion and amplitude of the longshore sand waves were not well reproduced. This particular capability has not yet been introduced into the release version of GENESIS and is considered an area for further study and model development.

### Cross-shore Seasonal Variation

A limitation of the standard 1-line concept is the lack of representation of sub-aqueous cross-shore transport, for example, to describe beach accretion under summer swell and erosion under shorter period and higher winter waves accompanying storms. As a long-term predictive technology focusing on alongshore processes, it is not consistent to resolve the impact of short-period storms, as it would require information about the profile shape. However, it may be possible to represent the aggregated effect of several storms and wave conditions to account for seasonal variations associated with cross-shore sediment transport. One approach to address this issue was presented in two conference papers (Hanson *et al.*, 1997; Hanson and Larson, 1998). A precondition for the formulation of the cross-shore contribution was that it should be compatible with the 1-line formulation in terms of independent variables and level of sophistication. The work was based on an analysis of an 11-year long time series of simultaneously collected data sets on waves and beach profiles from the US Army Corps of Engineers' Field Research Facility at Duck, North Carolina (Lee and Birkemeier, 1993) that were analyzed to investigate a possible relationship between the incident waves and the seasonal shoreline variations over a longer time period.

Because a 1-line model does not require or provide information about the time-varying shape of the beach profile, the cross-shore transport must be calculated using a relation that is independent of the profile shape. Numerous formulae for the cross-shore sand transport rate may be found in the literature (Horikawa 1988, p. 196 ff.). Many of these relations may be written in the generic form:

$$\frac{q_o}{wd} = K_q (\Psi - \Psi_c)^a \quad (8)$$

where  $q_o$  = cross-shore transport rate per unit width,  $w$  = sediment fall speed,  $d$  = grain size,  $K_q$  = transport coefficient,  $\Psi$  = Shield parameter,  $\Psi_c$  = critical Shield parameter, and  $a$  = empirical exponent. The value of  $K_q$  varies significantly

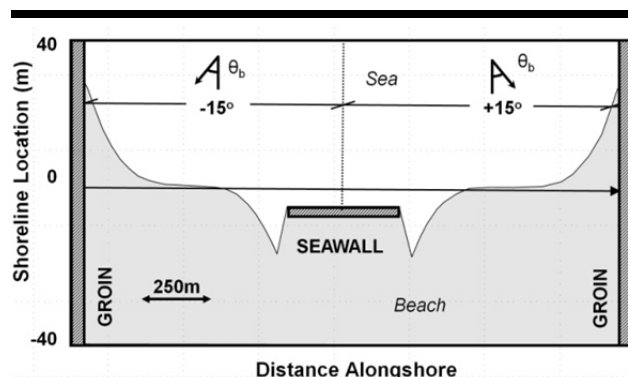


Figure 7. Hypothetical example of effect of seawall in the GENESIS model. (Modified from Hanson and Kraus, 1985)

between the different formulae. In a typical study, the value of this coefficient is determined in a calibration procedure. The value of the exponent  $a$  varies between 1 and 3 in the formulae reported in Horikawa (1988). Here the value  $a = 1.5$ , as proposed by Watanabe (1982), was used. For simplicity,  $\Psi_c$  was set to zero. To be applicable to the 1-line concept, the transport rate was assumed to be uniformly distributed across the surf zone.

The Shield parameter may be written in the form

$$\Psi = \frac{f_w u^2}{2sgd} = K_\Psi \frac{u^2}{gd} \quad (9)$$

where  $f_w$  = Jonsson (1966) wave friction factor,  $u$  = maximum horizontal wave-induced fluid velocity at the bottom,  $s$  = sediment specific density in water,  $g$  = acceleration due to gravity, and  $K_\Psi$  = friction coefficient. In addition, at wave breaking, the maximum fluid velocity may be approximated as  $u^2 \sim g H_b \sim g H_o$  (Kaminsky and Kraus, 1994), where  $H_b$  = breaking wave height, and  $H_o$  = deep-water wave height. Thus, the cross-shore sediment transport rate per unit length alongshore,  $q_o$ , here regarded as a potential rate, may be calculated, with  $K = K_q K_\Psi$

$$q_o = K_q K_\Psi w d \left( \frac{u^2}{gd} \right)^{3/2} = K w d \left( \frac{H_o}{d} \right)^{3/2} = K w \left( \frac{H_o^3}{d} \right)^{1/2} \quad (10)$$

Equation (10) gives only the potential magnitude of the transport rate and not the direction. Kraus *et al.* (1991) examined the capability of several criteria for predicting the direction of cross-shore sediment transport. One of these criteria was based on the non-dimensional fall speed  $N_o = H_o / (wT)$ , also known as the Dean number (Dean 1973), where  $T$  = peak wave period. The study identified a critical value  $N_c = 3.2$ , for which the following limiting values were found (for field, not laboratory conditions):

$$\begin{aligned} N_o < 0.75 N_c = 2.4 & \Rightarrow \text{onshore transport} \\ N_o > 1.25 N_c = 4.0 & \Rightarrow \text{offshore transport} \end{aligned} \quad (11)$$

For values between the two limits, the direction of transport was ambiguous.

The same criterion was adopted to represent shoreline movement by cross-shore processes, with the critical value  $N_c$  that separates onshore from offshore transport determined in the calibration procedure. Based on the wave time series, the potential cross-shore sediment transport rate,  $q_o$ , was calculated at each time step according to Eq. (10). At the critical value  $N_o = N_c$  the cross-shore transport rate was set to zero. From here, the actual transport  $q$  was assumed to increase linearly to  $+q_o$  at  $N_o = 0.75 N_c$ , where the plus sign indicates onshore transport or accretion. For smaller values of  $N_o$ ,  $q$  remained at  $+q_o$ . For erosional wave conditions,  $q$  was assumed to decrease linearly to

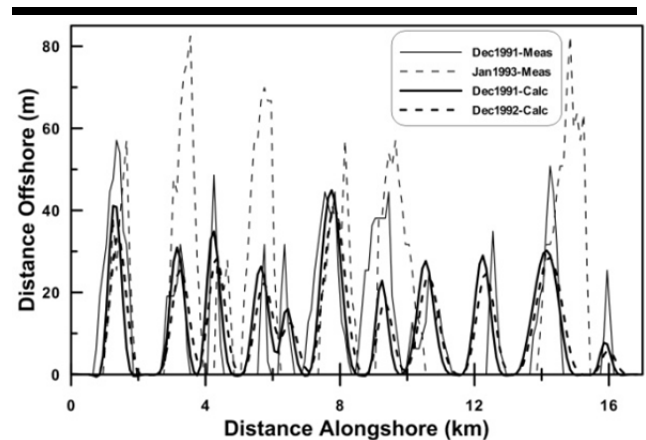


Figure 8. Measured and calculated longshore sand wave movement at Southampton Beach, LI, NY. Legend format = YYMM. M = measured. C = calculated. (Modified from Hanson *et al.*, 1996)

$-q_o$  at  $N_o = 1.25 N_c$ , where the minus sign indicates offshore transport. For larger values of  $N_o$ ,  $q$  remained at  $-q_o$ .

Shoreline location  $y$  was calculated based on the continuity relation as given by Eq. (4). By assuming no longshore transport gradients ( $\partial Q / \partial x = 0$ ), the shoreline change  $\Delta y$ , during a single time step,  $\Delta t$ , is given by:

$$\Delta y = \pm q \frac{\Delta t}{D} \quad (12)$$

where a positive sign corresponds to onshore transport. The best fit value of  $N_c$  was determined to be 3.8, which is in good agreement with the data presented in Kraus *et al.* (1991). Figure 9 compares the calculated shoreline variation and the actual change relative to the linear trend filtered with a moving average. Although there are some differences, the general behavior of the shoreline seems to be well reproduced. Hanson and Larson (1998) extended the study to describe random waves.

This capability to represent cross-shore transport was not introduced in the release version of GENESIS, but it is presently incorporated into GenCade (Hanson *et al.*, 2011) – a successor model of GENESIS.

### Sediment Transport by Tidal Currents

GENESIS has been widely applied for prediction of long-term shoreline evolution along wave-dominated open-coast beaches. Significant offsets between up- and down-drift beaches are typically found adjacent to inlet jetties where a dominant direction of longshore sediment transport exists. Down-drift beaches often suffer from chronic erosion and are in need of remedial measures. However, a quantitative tool was found lacking for developing, designing, and comparing the functioning of such proposed solutions. In its original version,

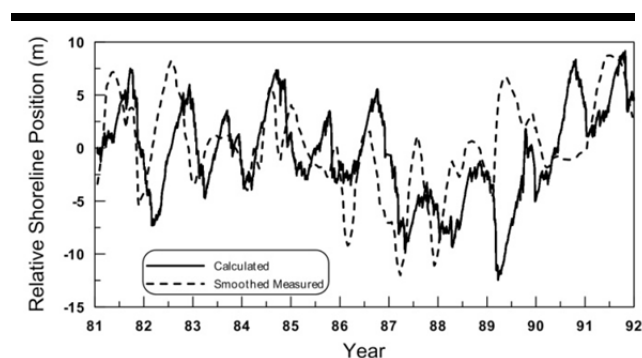


Figure 9. Comparison between calculated and measured shoreline positions at Duck, NC. (Modified from Hanson and Larson, 1997)

GENESIS calculated shoreline response for a wide variety of coastal features and engineering activities, under the assumption that wave-generated currents dominate the longshore sediment (typically sand or sand-sized particles) transport. It became evident, though, that the assumption of breaking waves as the sole mechanism for longshore sediment transport may not be sufficient in the vicinity of coastal inlets where tide- and sometimes wind-generated currents can play a significant role on the beaches adjacent to the inlet, especially if the inlet is not stabilized by jetties or is stabilized by small or highly permeable jetties.

An initial simplified attempt to represent tidal currents and longshore sediment transport in GENESIS was presented by Hanson *et al.* (2001). Here, the formulation was based on a simplified version of the Bagnold (1963) approach. Later, in Hanson *et al.* (2006), a more complex derivation was made. Larson and Hanson (1996) and Larson and Bayram (2005) developed a longshore sediment transport formula for the surf zone based on the hypothesis that wave breaking stirs up sand and maintains an average concentration distribution  $c(y, z)$  (*i.e.*, though the energy dissipation) in the surf zone, where  $c$  is volumetric concentration, and transported by the mean current. It was assumed that the total amount of work,  $W_s$ , necessary to keep the sand in suspension at steady-state is:

$$W_s = \int_0^{y_b} \int_0^h c(y, z)(\rho_s - \rho)gwdzdy \quad (13)$$

where  $y_b$  = width of the surf zone,  $z$  = vertical coordinate, and  $h$  = water depth. The wave energy flux that approaches to the shore is  $F_b \cos \theta_b$  and a certain portion,  $\varepsilon$ , of this is used for the work  $W_s$ ; therefore,  $W_s = \varepsilon F_b \cos \theta_b$ . If the longshore sand transport rate is the product of the local concentration and the longshore current velocity  $V$ , which is taken to be constant, then integrating the longshore transport across the profile to obtain the total transport rate yields:

$$Q = \frac{\varepsilon}{(\rho_s - \rho)(1 - p)gw} F_b \cos \theta_b \bar{V} \quad (14)$$

where  $Q$  = total longshore transport rate, and  $\bar{V}$  = mean longshore current. The value of  $\varepsilon$  was estimated by Bayram *et al.* (2006) through comparison with extensive field and laboratory data to be,

$$\varepsilon = \left( 9.0 + 4.0 \frac{H_b}{wT} \right) \cdot 10^{-5} \quad (15)$$

It is further hypothesized that different longshore currents whatever the origin, for example, those produced by tide or by wind, may be linearly superimposed to form a total mean longshore current  $\bar{V}$  responsible for transporting sand alongshore according to Eq. (14). Thus, the following relationship for the total longshore transport rate was derived as:

$$Q = \frac{\varepsilon}{(\rho_s - \rho)(1 - p)gw} F_b \cos \theta_b \cdot \left[ K_1 \left( \frac{5}{32} \frac{\pi \gamma}{c_f} \sqrt{g} A^{3/2} \sin \theta_b + \bar{V}_t + \bar{V}_w \right) - K_2 \left( \frac{\pi}{c_f \gamma^2} \sqrt{g h_b} \frac{\partial H_b}{\partial x} \right) \right] \quad (16)$$

where  $\gamma = H_b/h_b$ ,  $A$  = sediment shape parameter for an equilibrium beach profile following Dean (1977), and  $c_f$  = bottom friction coefficient. The  $A$  parameter was taken as  $A = 9/4 (w^2/g)^{1/3}$  after Kriebel *et al.* (1991), and  $c_f$  was given a

default value of 0.005.  $\bar{V}_t$  is the surf-zone average longshore tidal current velocity, and  $\bar{V}_w$  is the surf-zone average longshore wind-induced current velocity. This hypothesis of superposition of longshore current components was tested against laboratory measurements as presented in Hanson *et al.* (2006). The experiments were conducted in the U.S. Army Corps of Engineers' Large-Scale Sediment Transport Facility (LSTF), a basin that is 30 m wide, 50 m long, and 1.4 m deep, that is designed with the capability of simulating conditions comparable to low-energy coasts. Five movable bed tests were conducted in the LSTF (Gravens and Wang, 2007), of which the first four were used for comparisons with GENESIS. Figure 10 shows a comparison for Test 2, where a constant current representing a tidal current was added to the wave-generated current. This was achieved by re-circulating 1.5 times the estimated wave-generated longshore flux of water by means of pumps. GENESIS used the LSTF model as a 'prototype' case setting all numerical model values equal to those in the lab model. Each run lasted for about 24 hours. As seen from the figure, the overall shoreline change was fairly well reproduced, although the tombolo calculated by GENESIS was wider than that in the physical model test.

### Regional Depth Contour

Unless run together with a two-dimensional external wave transformation model, the offshore contour orientation in



GENESIS upon which the incoming waves are refracted is represented as a smoothed rendering of the shoreline orientation. This rendering is to assure that the incident waves are realistic while preserving feedback between shoreline change and the wave transformation. However, the methodology has a limitation: an open coast without structures or sources or sinks of sediment will evolve to a straight line if a standard shoreline response model is run a sufficiently long time. This limitation can be remedied by specifying a fixed representative contour (Hanson *et al.* 2001), which is appended to the feed-back contour associated with local changes in the shoreline. Correctly specified, the waves transformed over this contour within GENESIS will maintain an observed overall shoreline curvature, e.g., preserving a bay shape without the presence of structures, even if the model is run for very long time periods.

In Larson *et al.* (2002, 2006) the procedure was brought one step further in the related Cascade model in that the regional contour orientation was used to transform the incoming waves. Thus, from a wave transformation perspective, the regional trend is subtracted from the shoreline and contour orientation. As a result, the shoreline will, on an open coast without structures, gradually evolve into the shape of the regional contour rather than into a straight line. This capability was transferred into GENESIS in 2002. Figure 11 illustrates the working of the pre-specified regional contour on the long-term evolution of a concave embayment with open lateral boundaries, but without any structures. Without the regional contour (panel a) the embayment will gradually fill, and the shoreline evolve into a straight line. However, with the regional contour (panel b), the shoreline evolution will be guided by the contour. Gradually, the shoreline will become parallel to the regional contour rather than to a straight line.

### Interaction between Beach Berm and Dune

Dunes and berms exchange sand. During storms, waves may reach the dunes and erode sand from them that will be provided to the berm and surf zone. In-between storms, onshore-directed wind gradually transports sand from the beach to the dunes. This exchange of sand between the berm and dune plays a central role in the long-term behavior of a beach and should, therefore, be included in long-term morphological modeling.

Westhampton is located on the south shore of Long Island, NY, between Shinnecock Inlet and Moriches Inlet. The groin field and beaches at Westhampton have been well studied, and considerable data are available for quantitative assessments (Nersesian *et al.* 1992; Hanson *et al.*, 2008). The groins functioned as intended in protecting a once-vulnerable 4.8-km long segment of barrier beach that had experienced repeated breaching. However, the groin field caused erosion of down-drift beach directly to the west. As a consequence, proposals have been put forward regarding shortening of the groins so that they would release some portion of the impounded sand from the beach as well as the dunes. A reconnaissance study was performed with GENESIS where not only the evolution of the beach berm was included, but also the beach and dune interaction, including storms, over a 50-year calculation interval. For that study, algorithms for the interaction between the berm and the dune were developed and introduced into a research

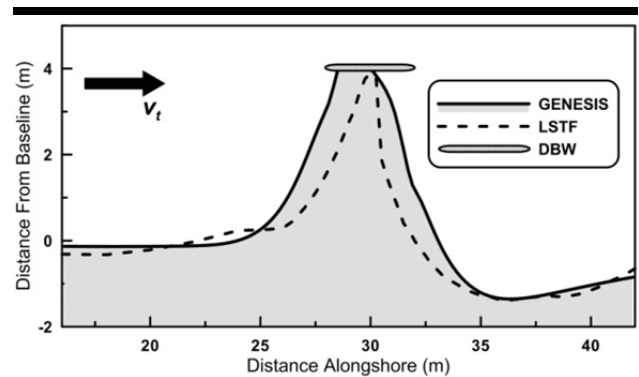


Figure 10. Calculated and measured final shorelines for LSTF Test 2. (Modified from Hanson *et al.*, 2006)

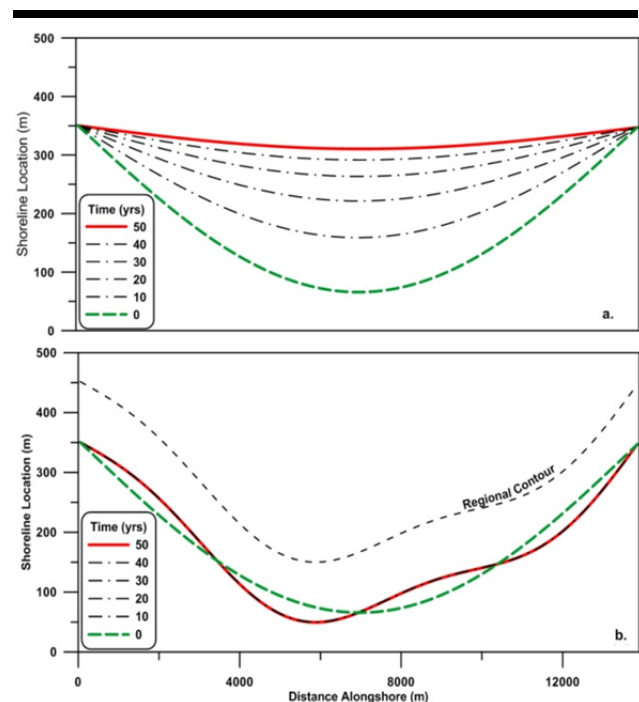


Figure 11. Long-term (50 years) simulations of concave embayment with open lateral boundaries with free influx of sediment. Panel a) illustrates situation without regional contour. Panel b) illustrates situation with regional contour.

version of GENESIS (Hanson *et al.* 2010).

Following the standard mass conservation relations for 1-line models, the governing equations for the movement of the dune toe  $y_D$  and the berm crest location  $y_B$  are  $dy_D/dt = (q_w - q_o)/D_D$  and  $dy_B/dt = -(q_w - q_o)/(D_B + D_C)$  respectively, where  $q_w$  = onshore sand transport to the dune by wind,  $q_o$  = erosion of the dune due to wave impact, and  $D_D$  = dune height.

It is assumed that sand transport to the dune is related to the width of the berm up to some distance over which equilibrium conditions have developed between the wind and the sand surface (Davidson-Arnott and Law, 1990; Davidson-Arnott *et al.*, 2005). A simple equation that exhibits these properties and at the same time gives a continuous description of the transport with changes in berm width, is:

$$q_w = q_{wo} \left( 1 - 0.5 \left( 1 - \tanh \left[ \frac{\pi}{q_{grad}} (y_B - y_D - y_{50}) \right] \right) \right) \quad (17)$$

where  $q_{wo}$  = maximum transport by wind for an infinitely wide beach, dependent on water and sand properties,  $y_B$  and  $y_D$  = distances to the seaward end of the berm and the dune toe, respectively, with the y-axis pointing offshore,  $y_{50}$  = distance from the seaward end of the berm to where the wind-blown transport has reached 50% of its maximum, and  $q_{grad}$  = transport gradient at  $y_{50}$ . The erosion rate due to dune impact by waves may be estimated as (Larson *et al.* 2004):

$$q_o = 4C_s \frac{(R + \Delta h - z_D)^2}{T}, \quad R > z_D - \Delta h \quad (18)$$

where  $R$  = run-up height (including setup) estimated from  $R = a\sqrt{H_o L_o}$ , in which  $H_o$  = deep-water root-mean-square wave height,  $L_o$  = deep-water wavelength, and  $a$  = coefficient (about 0.15, which corresponds to a representative foreshore slope);  $\Delta h$  = total water level (surge plus tide elevation relative to mean sea level, MSL);  $z_D$  = dune toe elevation (with respect to MSL);  $T$  = swash period (taken to be the same as the wave period); and  $C_s$  = empirical coefficient. In the numerical implementation,  $q_o$  varies at each time step and is computed from the input time series of waves. Placing cross-shore sediment transport contributions within a numerical 1-line model context, the mass conservation equation becomes,

$$\frac{\partial y}{\partial t} = -\frac{1}{D_B + D_C} \left( \frac{\partial Q}{\partial x} \right) + \frac{\partial y_B}{\partial t} = -\frac{1}{D_B + D_C} \left( \frac{\partial Q}{\partial x} - q_o + q_w \right) \quad (19)$$

To illustrate the interaction between longshore and cross-shore transport processes, an example is given for a straight shoreline and longshore transport generated using the U.S. Army Corps of Engineers' Wave Information Study hindcast time series for Westhampton Beach. The simulation covers the 15-year period July 1, 1980 to June 30, 1995. A constant gradient was imposed on the longshore transport such that the beach would erode when the transport direction was positive (to the west) and accrete for negative transport rates. The simulation results are illustrated in Figure 12, where the shoreline evolution with only longshore processes active is shown as a solid line. The calculated net shoreline change was 7.5 m corresponding to 0.5 m/yr, which is a realistic number for this site. Because cross-shore transport was not included in this example, the location of

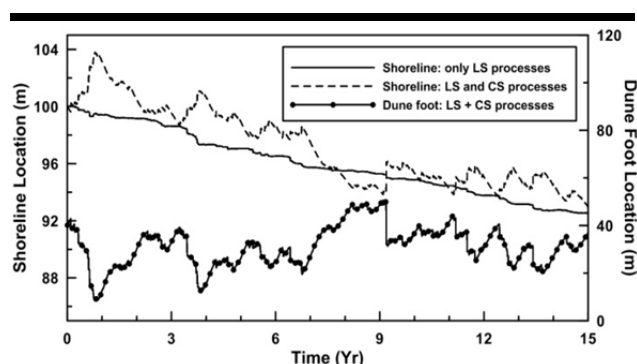


Figure 12. Shoreline change over 15 years with only longshore (LS) processes included and both LS and cross-shore (CS) processes included, respectively.

the dune toe did not move and is not shown.

If cross-shore transport is represented in GENESIS (by including wind-blown sand and dune erosion in the model application), the dune volume varies which, in turn, induces increasing shoreline fluctuations as material exchange occurs between the dune and the shoreline (Figure 12, dashed line). The long-term longshore trend is still apparent with a shoreline net change trend of 6.7 m over the 15 years. As seen from Eq. (19), shoreline and dune toe fluctuations are coupled. For Westhampton Beach, dune height  $D_D$  is on the average 1.5 m, the berm height  $D_B$  is 3 m, and the depth of closure  $D_C$  is 8 m. Thus, dune fluctuation will be scaled as 11/1.5 times the shoreline fluctuations, as seen from Figure 12. The calculated example shows that the interaction between the dune and the berm functions as intended.

## SUMMARY AND CONCLUSIONS

GENESIS has been in existence for some 25 years. It started as a traditional 1-line model, and it has been continuously enhanced to include novel features not previously introduced in 1-line models. The objective of these enhancements has been to meet the needs and challenges that have been called for through hundreds of engineering applications and equally many users. This paper outlined the most significant of these developments, while having to omit numerous others. Enhancements discussed here were: line sources and sinks of sand, sand transport near shore-normal structures including natural bypassing, wave transmission through and shoreline response to shore-normal structures, seawalls, migrating longshore sand waves, preservation of the regional shape of the shoreline, seasonal variation by cross-shore sand transport, sand transport by tidal currents and wind-generated currents, and the interaction between the beach berm and the dunes behind it. The development has been done in a consistent way, based on literature reviews, beta testing, comparison to beach behavior, and quality control. The challenge has been not only to represent the features themselves, but also to be consistent to the basic assumptions of shoreline modeling theory.

One feature that has contributed to wide applicability of GENESIS that could not be discussed here is the generalized interface that was developed originally in the mid 1980s. Through this feature, the model can easily be applied in a wide range of different locations and engineering configurations, allowing applications by users outside the group of developers. The interface was a major contribution to the fact that GENESIS was quickly adopted by a large number of consultants, academics, and students world wide. In addition, the use of the model was greatly facilitated by the analysis tools and utility programs supporting GENESIS applications documented in Gravens *et al.* (1991) and Gravens (1992). Subsequently it was incorporated into a modern Windows-based integrated modeling system known as NEMOS (Nearshore Evolution Modeling System) and distributed as part of the CEDAS (Coastal Engineering Design and Analysis System) software package.

Through these added capabilities, GENESIS has evolved to meet the challenges of modern, multi-scale, long-term coastal engineering applications.

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